Section 4
DESIGN AND ANALYSIS PROCEDURES (SDAP)

4.1 SINGLE SPAN BRIDGES

For single-span bridges, regardless of seismic zone and in lieu of a rigorous analysis, the minimum design force at the connections in the restrained direction between the superstructure and the substructure shall not be less than the product of $F_a S_S / 2.5$, and the tributary permanent load.

4.2 SDAP A1 AND A2

The design requirements for SDAP A1 and A2 are specified in Section 6. There are no dynamic analysis requirements specified for SDAP A1 and A2

4.3 SDAP B — NO SEISMIC DEMAND ANALYSIS

Bridges qualifying for SDAP B do not require a seismic demand analysis but capacity design principles and minimum design details are required. The capacity design forces are covered in more detail in Article 4.8.

4.3.1 No Analysis Approach

SDAP B consists of the following steps:

- Step 1 - Check Article 4.3.2 for restrictions on structural and site characteristics to determine if SDAP B is applicable. The bridge site must not exceed $F_v S_1$ limitations and the structure must meet certain regularity requirements as defined in Section 4.3.2.
- Step 2 - Reinforced concrete columns shall be designed using non-seismic loading cases and checked for minimum longitudinal reinforcement (0.8%).
- Step 3 - Reinforced concrete columns shall be detailed to meet the shear, confinement and bar restraint reinforcement requirements of Article

7.8.2.3 through 7.8.2.6 in the plastic hinge zones defined in Article 4.9.
- Step 4 - Steel columns shall be designed using non-seismic loading cases and checked for minimum width to thickness ratios as described in Article 7.7.4. Plastic hinge zone forces shall be those from capacity design procedures of Article 4.8.
- Step 5 -Members connecting to columns shall be designed to resist column plastic moments and shears using the principles of capacity design described in Article 4.8.
- Step 6 - Foundations (soils and piles) shall be designed to resist column moment and shears using the principles of capacity design described in Article 4.8 using an overstrength ratio of 1.0 for all columns.

4.3.2 Limitations

SDAP B shall be used only at sites where:

$$F_v S_1 < 0.4 \cos \alpha_{skew} \qquad (4.3.2\text{-}1)$$

where α_{skew} = the skew angle of the bridge, (0 degrees being the angle for a right bridge).

Additionally, SDAP B shall be used only on structures that comply with the following restrictions:

For concrete column and pile bents

- $P_e < 0.15 f_c' A_g$
- $\rho_\ell > 0.008$
- $D > 300mm$ (12 inches)
- $\dfrac{M}{VD} < 6$

where P_e = column dead plus seismic live load

f_c' = nominal 28 day concrete strength

A_g = gross cross-sectional area of column

ρ_ℓ = longitudinal reinforcement ratio
D   = column transverse dimension
M   = maximum column moment
V   = maximum column shear

For concrete wall piers with low volumes of longitudinal steel:

- $P_e < 0.1 f'_c A_g$

- $\rho_\ell > 0.0025$

- $\dfrac{M}{Vt} < 10$

- $t > 300 mm\,(12\,inches)$

where t = wall thickness, or smallest cross-sectional dimension.

For steel pile bents framing into reinforced concrete caps:

- $P_e < 0.15 P_y$

- $D_p \geq 250 mm\,(10\,inches)$

- $L/b < 10$

where D_p = pile dimension about the weak axis bending at ground line.

    b = flange width or pipe diameter

    P_y = axial yield force of steel pile

    L = length from the point of maximum moment to the inflexion point of the column when subjected to a pure transverse load.

For timber piles framing into reinforced concrete caps or steel moment-frame columns:

- $P_e < 0.1 P_c$

- $D_p \geq 250 mm\,(10\,inches)$

- $\dfrac{M}{VD_p} < 10$

where P_c =   axial compression capacity of the pile.

SDAP B shall NOT be used for bridges where:

- Individual interior bent stiffnesses vary by more than a factor of 2 with respect to the average bent stiffness of the bridge.
- The maximum span exceeds 80 m.
- The maximum span length is more than 50 percent longer than the average span length.
- The maximum skew angle exceeds 30 degrees
- For horizontally curved bridges the subtended angle exceeds 30 degrees.
- For frames in which the superstructure is continuous over the bents and for which some bents do not participate in the ERS, $F_v S_1$ factored by the ratio of the total number of bents in the frame divided by the number of bents in the frame that participate in the ERS in the longitudinal direction exceeds $0.4 \cos \alpha_{skew}$
- If the bridge site has a potential for liquefaction and the piers are seated on spread footings.
- The bridge site has a potential for liquefaction and the piers are seated on piled foundations unless the piles shall be detailed for ductility, in accordance with these provisions over the length passing through the liquifiable soil layer plus an additional length of three-pile diameters or 3 m (10 ft) whichever is larger, above and below the liquefiable soil layer.

4.3.3 Capacity Design and Strength Requirements of Members Framing into Columns

Except for the geotechnical design of foundations, SDAP B requires the use of capacity design for all components connected to the columns (Article 4.8). For the geotechnical design of foundations, the moment overstrength capacity of columns that frame into the foundations need not be taken as greater than:

$M_{po} = 1.0\, M_n$

where

M_{po} = plastic overstrength capacity of a column

M_n = nominal moment capacity of a column

4.4 SDAP C — CAPACITY SPECTRUM DESIGN METHOD

4.4.1 Capacity Spectrum Design Approach

SDAP C combines a demand and capacity analysis, including the effect of inelastic behavior of ductile earthquake resisting elements. The procedure applies only to bridges that behave essentially as a single degree-of-freedom system. SDAP C is restricted to bridges with a very regular configuration as described in Article 4.4.2 and with the recommended earthquake resisting systems (ERS) as described in Article 3.3.1.

The major steps in applying the capacity spectrum method for the two levels of earthquake are as follows:

- *Step 1* - Design the bridge for the non-seismic load combinations. Determine the applicability of SDAP C.
- *Step 2* - Check if the design for non-seismic loads satisfies the requirements for the 50% PE in 75-year earthquake event.
- *Step 3* - Design for the 50% PE in 75-year earthquake event if necessary from Step 2.
- *Step 4* - With a design that satisfies the non-seismic load combinations and the 50% PE in 75-year earthquake event, check that the requirements for the 3% PE in 75 year or 1.5 mean deterministic earthquake event are satisfied.
- *Step 5* - If necessary from Step 4, modify the design to satisfy the requirements for the 3% PE in 75-year or 1.5 mean deterministic event.
- *Step 6* - Design and detail the columns, the connections of the columns to the foundation, and superstructure or column bent using the capacity design procedures of Article 4.8. For bridges in SDR 3, the requirements of Article 4.3.3 are applicable. If the connection capacity design forces are excessive, then SDAP D shall be used to determine the elastic connection forces.

Details for each of these steps are discussed in the Commentary.

4.4.2 Limitations

SDAP C shall only be used on bridges that satisfy the following requirements:

- The number of spans per frame or unit shall not exceed six.
- The number of spans per frame or unit shall be at least three, unless seismic isolation bearings are utilized at the abutments.
- Abutments shall not be assumed to resist significant seismic forces in the transverse or longitudinal directions.
- Span length shall not exceed 60 m (200 feet).
- The ratio of span lengths in a frame or unit shall not exceed 1.5.
- Pier wall substructures must have bearings that permit transverse movement.
- The maximum skew angle shall not exceed 30 degrees, and skew of piers or bents shall not differ by more than 5 degrees in the same direction.
- For horizontally curved bridges, the subtended angle of the frame of all the bridge types shall not exceed 20 degrees.
- The ratio of bent or pier stiffness shall not vary by more than 2 with respect to the average bent stiffness, including the effect of foundation stiffness.
- The ratio of lateral strength (or seismic coefficient) shall not exceed 1.5 of the average bent strength.
- For concrete columns and pile bents:
  - $P \leq 0.20 f_c' A_g$
  - $\rho_\ell > 0.008$
  - $D \geq 300\text{mm}$ (12 inches)
- When liquefaction potential is determined to exist according to the requirements in Article 7.6 or 8.6, the piers or bents must have pile foundations.

Bridges that satisfy the above and have elastomeric, sliding, or isolation bearings at each pier and abutment shall use the provisions of Article 5.4.1.1.

4.5 SDAP D — ELASTIC RESPONSE SPECTRUM METHOD

SDAP D is a one step design procedure using an elastic (cracked section properties) analysis. Either the Uniform Load or Multimode method of

analysis may be used as per Article 5.4.2. The analysis shall be performed for the governing design spectra (either the 50% PE in 75-year or the 3% PE in 75-year or 1.5 mean deterministic) and the R-Factors given in Tables 4.7-1 and 4.7-2 shall be used to modify elastic response values. The analysis shall determine the elastic moment demand at all plastic hinge locations in the columns. Capacity design principles shall be used for column shear design and the design of all column connections and foundation design. If sacrificial elements are part of the design (i.e. shear keys) they shall be sized to resist the 50% PE in 75-year forces and the bridge shall be capable of resisting the 3% PE in 75-year or 1.5 mean deterministic forces without the sacrificial elements (i.e. two analyses are required if sacrificial elements exist in a bridge).

This design procedure consists of the following steps:

- *Step 1* - Design the bridge for non-seismic loading conditions.
- *Step 2* - Perform an elastic dynamic analysis as described in Article 5.4.2 for the 3% PE in 75-year or 1.5 mean deterministic earthquake loading to determine displacement demands. Analysis shall reflect the anticipated condition of the structure and the foundation during this earthquake.
- *Step 3* - Determine controlling seismic design forces for the moment design of all columns from an elastic dynamic analysis using either the 50% PE in 75- or 3% PE in 75-year or 1.5 mean deterministic earthquake. Analyses shall reflect the anticipated condition of the structure and the foundation during each of these earthquakes. Elastic forces from the analyses shall be modified using the appropriate R-Factors from Tables 4.7-1 and 4.7-2.
- *Step 4* – Check the minimum design base shear for each column using the *P-Δ* requirements from Article 7.3.4 or 8.3.4 using the elastic displacements obtained in Step 2. Modify column design as necessary.
- *Step 5* - Determine the design forces for other structural actions using capacity design principles as described in Article 4.8.

- *Step 6* - Design sacrificial elements to resist forces generated by the 50% PE in 75-year earthquake.

4.6 SDAP E — ELASTIC RESPONSE SPECTRUM METHOD WITH DISPLACEMENT CAPACITY VERIFICATION

SDAP E requires an elastic (cracked section properties) response spectrum analysis for the governing design spectra (50% PE in 75-year or 3% PE in 75-year/1.5 mean deterministic) and *P-Δ* design check. The results of these analyses shall be used to perform preliminary flexural design of plastic hinges in columns and to determine the displacement of the structure. To take advantage of the higher R-Factors in Table 4.7-1, displacement capacities shall be verified using two-dimensional nonlinear static (pushover) analyses in the principal structural directions. Design forces on substructure elements may be reduced below those obtained for the 3% PE in 75-year event or 1.5 mean deterministic divided by the R-Factor, as described in Step 2 below. Capacity design principles of Article 4.8 shall be used to design the connection of the columns to the superstructure and foundation and for column shear design. SDAP E is required when owner approved ERE are used that have inelastic action that cannot be inspected.

This design procedure shall consist of the following steps:

- *Step 1* - Perform Steps 1 through 4 for SDAP D except that the appropriate R-Factors from Tables 4.7-1 and 4.7-2 shall be used.
- *Step 2* - Perform a displacement capacity verification analysis using the procedures described in Article 5.4.3. If sufficient displacement capacity exists the substructure design forces may be further reduced from those of Step 1, but not less than 70% of the Step 1 forces nor less than design forces from the 50% PE in 75-year event. If column sizes are reduced, repeat Step 2 of SDAP D and these displacements shall be used in a repeat of this step in SDAP E.
- *Step 3* - Perform Steps 5 and 6 for SDAP D.

Table 4.7-1   Base Response Modification Factors , R$_B$, for Substructure

Design Earthquake	Substructure Element	Performance Objective Life Safety SDAP D	Performance Objective Life Safety SDAP E
MCE	Wall Piers – larger dimension	2	3
	Columns – Single and Multiple	4	6
	Pile Bents and Drilled Shafts – Vertical Piles – above ground	4	6
	Pile Bents and Drilled Shafts – Vertical Piles – 2 diameters below ground level-No owners approval required.	1	1.5
	Pile Bents and Drilled Shafts – Vertical Piles – in ground - Owners approval required.	N/A	2.5
	Pile Bents with Batter Piles	N/A	2
	Seismically Isolated Structures	1.5	1.5
	Steel Braced Frame – Ductile Components	3	4.5
	Steel Braced frame – Nominally Ductile Components	1.5	2
EE	All Elements for Expected Earthquake	1.3	1.3

Notes:

1. The substructure design forces resulting from the elastic analysis divided by the appropriate R-Factor for SDAP E cannot be reduced below 70% of these R-Factor reduced forces or the 50% PE in 75 year design forces as part of the pushover analysis.
2. There maybe design situations (e.g architecturally oversized columns) where a designer opts to design the column for an R=1.0 (i.e. elastic design) – Article 4.10. In concrete columns the associated elastic design shear force may be obtained from the elastic analysis forces using an R-Factor of 0.67 or by calculating the design shear by capacity design procedures using a flexural overstrength factor of 1.0. In steel braced frames if an R=1.0 is used the connection design forces shall be obtained using an R=0.67. If an R=1.0 is used in any design the foundations shall be designed for the elastic forces plus the SDR 2 detailing requirements are required for concrete piles. (i.e. minimum shear requirements) – Article 4.10.
3. Unless specifically stated, the R-Factors apply to both steel and concrete.
4. N/A in this case means that owners approval is required and thus SDAP E is required to use this design option.

Table 4.7-2   Response Modification Factors, R — Connections

Connection	All Performance Objectives
Superstructure to abutment	0.8
Expansion joints within a span of the superstructure	0.8
Columns, piers, or pile bents to cap beam or superstructure	0.8
Columns or piers to foundations	0.8

Note: These factors are not intended for those cases where capacity design principles are used to develop the design forces to design the connections.

4.7   RESPONSE MODIFICATION FACTORS

Structures that are designed using SDAP D or E shall use the response modification factors defined in this article.

To apply the R-Factors specified herein, the structural details shall satisfy the provisions of Articles 7.7 and 7.8 or 8.7 and 8.8 as appropriate.

Except as noted herein, seismic design force effects for flexural design of the primary plastic hinges in substructures shall be determined by dividing the force effects resulting from elastic analysis by the appropriate R-Factor, R, as given by

$$R = 1 + (R_B - 1)\frac{T}{1.25T_s} \leq R_B \qquad (4.7\text{-}1)$$

where R_B is given in Table 4.7-1., T is the period of vibration of the bridge, and T_s is defined in Figure 3.4.1-1.

The period T and resulting R-Factor from Equation 4.7-1 may be applied separately in the longitudinal and transverse directions provided there is no significant skew or curvature and the fundamental modes in the longitudinal and transverse directions have no significant coupling. If significant skew and curvature does exist, the lowest period T shall be used to determine the R-Factor in both the longitudinal and transverse directions.

4.8 CAPACITY DESIGN

Capacity design principles require that those elements not participating as part of the primary energy dissipating system (flexural hinging in columns), such as column shear, joints and cap beams, spread footings, pile caps and foundations be "capacity protected". This is achieved by ensuring the maximum moment and shear from plastic hinges in the columns (overstrength) can be dependably resisted by adjoining elements.

Exception: Elastic design of all substructure elements (Article 4.10), seismic isolation design (Article 7.10 and 8.10) and in the transverse direction of a column when a ductile diaphragm (Article 7.7.8 or 8.7.8) is used.

4.8.1 Inelastic Hinging Forces

Inelastic hinges shall form before any other failure due to overstress or instability in the structure and/or in the foundation. Except for pile bents and drilled shafts, and with owners' approval, inelastic hinges shall only be permitted at locations in columns where they can be readily inspected and/or repaired.

Superstructure and substructure components and their connections to columns that are designed not to yield shall be designed to resist overstrength moments and shears of yielding members. Except for the geotechnical aspects of design of foundations in SDR 3, the moment overstrength capacity (M_{po}) of column/pier/pile members that form part of the primary mechanism resisting seismic loads shall be assessed using one of the following approaches:

- $M_{po} = \lambda_{mo} M_n$ where
  λ_{mo} = 1.5 for concrete columns
     = 1.2 for steel columns
     = 1.3 for concrete filled steel tubes
     = 1.5 for steel piles in weak axis bending and, for steel members in shear (e.g. eccentrically braced frames),
     = 1.0 for geotechnical design forces in SDR 3

  where M_n is the nominal moment strength in which expected yield strengths are used for steel members (Articles 7.7.2 and 8.7.2) and λ_{mo} is the overstrength factor.
- For reinforced concrete columns the plastic analysis approach given by Article 7.8.2.8.
- For reinforced concrete columns a compatibility section analysis, taking into account the expected strengths of the materials and the confined concrete properties and the strain hardening effects of the longitudinal reinforcement.

These overstrength moments and associated shear forces, calculated on the basis of inelastic hinging at overstrength, shall be taken as the extreme seismic forces that the bridge is capable of developing. Typical methods of applying capacity design at a bent in the longitudinal and transverse directions are shown in Figure 4.8.1-1.
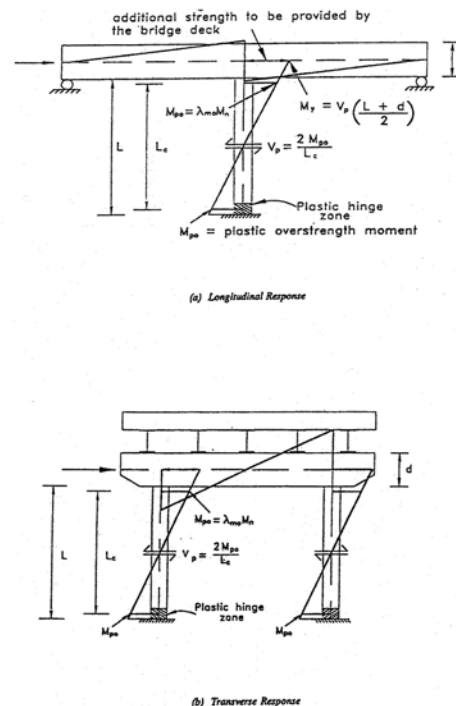

Figure 4.8.1-1   Capacity Design of Bridges Using Overstrength Concepts

4.8.1.1 Single Column and Piers

Column shear forces and design moments in the superstructure, bent caps, and the foundation structure shall be calculated for the two principal axes of a column and in the weak direction of a pier or bent as follows:

- *Step 1*. Determine the column overstrength moment capacities. Use an overstrength factor given in Article 4.8.1 times the nominal moment. The nominal moment for steel members is calculated using the expected yield strengths of Article 7.7.2. For both materials use the maximum elastic column axial load from Article 3.6 added to the column dead load. Column overstrength moments should be distributed to the connecting structural elements. (Exception: when calculating the design forces for the geotechnical aspects of foundations in SDR 3, use an overstrength factor of 1.0 on the nominal moment.)
- *Step 2*. Using the column overstrength moments, calculate the corresponding column shear force assuming a quasi-static condition. For flared columns designed to be monolithic with the superstructure or with isolation gaps less than required by Article 7.8.2 or 8.8.2, the shear shall be calculated as the greatest shear obtained from using:

  a. The overstrength moment at both the top of the flare and the top of the foundation with the appropriate column height.

  b. The overstrength moment at both the bottom of the flare and the top of the foundation with the reduced column height.

If the foundation of a column is more than 1-column diameter below ground level, the column height for the capacity shear force calculation shall be based on the mud or ground line, not the top of the foundation.

For pile bents or drilled shafts, the length of the pile or drilled shaft shall be not lower than the ground line for the purpose of calculating the capacity design shear force.

The forces corresponding to a single column hinging are:

- Axial Forces —unreduced maximum and minimum seismic axial load of Article 3.6 plus the dead load.
- Moments—those calculated in Step 1. Exception: An overstrength factor of 1.0 is required for geotechnical design forces in SDR 3.
- Shear Force—that calculated in Step 2.

4.8.1.2 Bents with Two or More Columns

The forces for bents with two or more columns shall be calculated both in the plane of the bent and perpendicular to the plane of the bent. Perpendicular to the plane of the bent the forces shall be calculated as for single columns in Article 4.8.1.1. In the plane of the bent the forces shall be calculated as follows:

- *Step 1*. Determine the column overstrength moments. Use overstrength factors given in Article 4.8.1 on the nominal strength calculated using the expected yield strength for structural steel. For both materials use the axial load corresponding to the dead load. Exception: When calculating the design forces for the geotechnical aspects of foundations in SDR 3 use an overstrength factor of 1.0 on the nominal moment.
- *Step 2*. Using the column overstrength moments calculate the corresponding column shear forces. Sum the column shears of the bent to determine the maximum shear force for the bent. If a partial-height wall exists between the columns, the effective column height is taken from the top of the wall. For flared columns and foundations below ground level see Article 4.8.1.1 - Step 2. For pile bents and drilled shafts, the length of pile from the pile cap to the mud or ground line shall be used to calculate the capacity design shear force.
- *Step 3*. Apply the bent shear force to the top of the bent (center of mass of the superstructure above the bent) and determine the axial forces in the columns due to overturning when the column overstrength moments are developed.
- *Step 4*. Using these column axial forces combined with the dead load axial forces, determine revised column overstrength moments. With the revised overstrength moments calculate the column shear forces and the maximum shear force for the bent. If the maximum shear force for the bent is not within 10% of the value previously

determined, use this maximum bent shear force and return to Step 3.

The forces in the individual columns in the plane of a bent corresponding to column hinging, are:
- Axial Forces—the maximum and minimum axial load is the dead load plus or minus the axial load determined from the final iteration of Step 3.
- Moments—the column overstrength plastic moments corresponding to the maximum compressive axial load specified in (1) with an overstrength factor specified in Article 4.8.1 Exception: An overstrength factor of 1.0 is required for geotechnical design forces in SDR 3.
- Shear Force—the shear force corresponding to the final column overstrength moments in Step 4 above.

4.8.1.3 Capacity Design Forces

Design forces for columns and pile bents shall be determined using the provisions of Article 4.8.1.1 or 4.8.1.2 or the elastic design forces specified in Article 4.10. Capacity design forces for pier walls in the weak direction shall be determined using the provisions of Article 4.8.1.1 and those in the strong direction using Article 4.10. The capacity design forces for the shear design of individual columns, pile bents or drilled shafts shall be those determined using Article 4.8.1.1 or 4.8.1.2 as appropriate. The capacity design forces for the connection of the column to the foundation, cap beam or superstructure shall be the axial forces, moments and shears determined using the provisions of Article 4.8.1.1 or 4.8.1.2. The bearing supporting a superstructure shall be capable of transferring the shear forces determined using the provisions of Article 4.8.1.1 or 4.8.1.2 in both the longitudinal and transverse directions as per Article 7.9 or 8.9. Sacrificial elements are designed for the elastic forces from the 50% PE in 75 year earthquake. The capacity design forces for superstructure design (Article 8.11) shall either be the elastic forces from the analysis or where appropriate the moments and shears from Article 4.8.1.1 or 4.8.1.2. The abutment forces associated with the superstructure design shall be the elastic forces from the analysis.

4.9 PLASTIC HINGE ZONES

Columns, piers, pile bents/caissons and piles that participate in the ERS will have plastic hinges occurring and special detailing in these zones is specified in Articles 7.7, 7.8, 8.7, and 8.8 as appropriate. The plastic hinge zones defined below cover the potential range of locations where a plastic hinge may occur.

4.9.1 Top Zone of Columns, Pile Bents and Drilled Shafts

For concrete and steel columns, pile bents and drilled shafts the plastic hinge zone at the top of the member is defined as the length of the member below the soffit of the superstructure for monolithic construction and below the soffit of girders or cap beams for bents. The plastic hinge zone length shall be the maximum of the following.

- One sixth of the clear height of a reinforced concrete column
- One eighth of the clear height of a steel column
- 450mm
- The following additional criteria shall determine the maximum plastic hinge length in reinforced concrete columns

$$D\left(\cot\theta + \tfrac{1}{2}\tan\theta\right) \qquad (4.9.1\text{-}1)$$

$$1.5\left(0.08\,M/V + 4400\varepsilon_y d_b\right) \qquad (4.9.1\text{-}2)$$

$$M/V\left(1 - M_y/M_{po}\right) \qquad (4.9.1\text{-}3)$$

where
D = transverse column dimension in direction of bending
θ = principal crack angle from Equation. 8.8.2.3-4
ε_y = yield strain of longitudinal reinforcement
d_b = longitudinal bar diameter
M = maximum column moment
V = maximum column shear
M_y = column yield moment
M_{po} = column plastic overstrength moment

For flared columns the plastic hinge zone shall extend from the top of the column to a distance equal to the maximum of the above criteria below the bottom of the flare.

4.9.2 Bottom Zone of a Column Above a Footing or Above an Oversized In-ground Drilled Shaft

The plastic hinge zone above the top of the footing of a column or a drilled shaft designed so that the maximum moment is above ground shall be the maximum of the items given in Article 4.9.1 unless the footing or the transition between in ground and above ground drilled shafts is below the ground level in which case it shall extend from the top of the footing or the transition between the two shafts to a distance above the mud or ground line equal to the maximum of the items given in Article 4.9.1.

4.9.3 Bottom Zone of Pile Bents and Drilled Shafts/Caissons

The plastic hinge zone at the bottom of a pile bent or a uniform diameter drilled shaft/caisson shall extend a distance above the mud or ground line equal to the maximum of the items specified in 4.9.1 to a distance 10D below the mud or ground line or 5 m (15 ft.) whichever is greater. It need not exceed 3D below the point of maximum moment in the ground. If scour or liquefaction may occur the plastic hinge zone as a minimum shall extend a distance of 3D below the mean scour depth or 3D below the lowest liquefiable layer. If a drilled shaft has an oversized in-ground shaft the top 10D of the oversized shaft shall be treated like the zone of a pile below the pile cap.

4.10 ELASTIC DESIGN OF SUBSTRUCTURES

There may be instances where a designer chooses to design all of the substructure supports elastically (i.e., R=1.0 for all substructures) or in some cases a limited number of substructure elements are designed elastically

4.10.1 All Substructure Supports are Designed Elastically

The elastic design forces for all elements are obtained from SDAP D using either an R=1.0 or 0.8 as specified in Table 4.7-2. The design force for any elements that could result in a brittle mode of failure (e.g., shear in concrete columns and pile bents, connections in braced frames) shall use an R-Factor of 0.67 with the elastic force. As an alternate to the use of the elastic forces, all elements connected to the column can be designed using the capacity design procedures of Article 4.8 using an overstrength ratio of 1.0 times the nominal moment capacities.

4.10.2 Selected Substructure Supports are Designed Elastically

If selected substructure supports are designed elastically then the moment demand can be established using an R=1.0 from the SDAP D analysis. The column or pile bent shear force and all connecting elements shall be designed using the capacity design procedures of Article 4.8 or the requirements of Article 4.10.1.

Exception: The component design procedures of Article 4.10.1 may be used, provided the SDAP D analytical model uses the secant modulus of columns that are not designed elastically. The secant stiffness (K_{sec}) of the columns shall be based on the elastic displacements from an iterated analysis as shown in Figure 4.10-1 where M_n is the nominal moment capacity and Δ_e is the elastic displacement of the bent as defined in Article 7.3.4 or 8.3.4
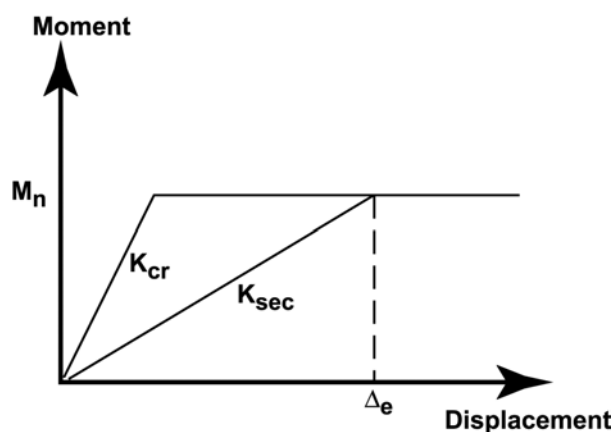

Figure 4.10-1 Characterization of the Secant stiffness of a column